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POTTER CREEK MASTER DRAINAGE PLAN

Prepared for:

QUINTE CONSERVATION
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1. INTRODUCTION

1.1 Background

The Potter Creek drainage basin straddles the boundary between the City of Belleville and the City of Quinte West. The basin is approximately 31 square kilometres and extends from the Bay of Quinte to Tuckers Corners north of highway 401. The eastern limits of the basin, in the old City of Belleville, have been developed for some time. Except for the Loyalist College development and strip development along highway 401, the remainder of the basin remains largely undeveloped. This situation is about to change as a result of development initiatives in the southeastern part of the basin.

Ecos-Garatech (1994) prepared a subwatershed plan for Potter Creek that, among other things, was intended to provide a) input to municipal land use plans and b) directions to proponents of development. The flood plain mapping section of the report provided, for various return periods up to the 100 years, sub basin peak flows and routed flows at various locations along the creek. The report also provided information on location and size of 12 stormwater management facilities. Recently, Marshall Macklin Monaghan (2006) prepared a master drainage plan and stormwater management plan for two tributaries of Potter Creek.

The City of Belleville is in the process of revising its official plan and Quinte West is in the final stages of preparing an update to its official plan. Given that the planned levels of future development are available, this is an opportune time to update the 1994 subwatershed plan for the Potter Creek drainage basin. Quinte Conservation (QC) decided to do a large portion of the updating work in-house, but to engage a specialist consultant to carry out the required stormwater hydrologic modelling and stormwater management studies. Accordingly, in January 2007, QC retained XCG Consultants Inc. to conduct hydrologic modelling studies in support of master drainage plan development. The scope of work and objectives are presented in the following sections.

1.2 Scope of Work

The scope of work is set out in a letter from Quinte Conservation to XCG Consultants Inc. dated January 17, 2007, in which QC requested assistance with some components of the design of the master drainage plan, such assistance to include

- **i.** performing a hydrology study to establish a basis for design of facilities and recommendations for other measures,
- **ii.** application of physical constraints as well as those imposed by planning regulations, policies and guidelines in proposing a strategy for maintaining watershed protection,
- **iii.** preliminary design (generic) of facilities including location, design and cost, and
- **iv.** preparation of a strategy for implementation including the staging and financing of facilities, maintenance and operation.

1.3 Objectives

The specific objectives of this work are as follows.

- **i.** For existing conditions, determine peak flows generated by the 12-h, 100-year storm rainfall at the outlets of all major sub basins and at all junctions in the stream network.

ii. For post-development conditions (as defined by official plans), determine peak flows at the same locations and for the same storm rainfall as set out in (i) above.

iii. At the outlets of all major sub basins, determine the storage required to reduce post-development peaks to existing condition peak flows.

iv. At the outlets of all major sub basins, determine values for water quality storage according to Ministry of Environment and Bay of Quinte Remedial Action Plan guidelines.

v. Provide guidance on basin-wide stormwater management measures.

vi. Provide a generic design of a typical stormwater management facility that will accomplish the water quantity and water quality objectives set out in iii and iv.

vii. Provide guidance on the apportionment of capital cost of a stormwater management facility that will serve a number of consecutive and/or concurrent developments.

viii. Provide guidance on operation and maintenance and associated costs.

2. HYDROLOGIC MODELING

2.1 Overview

2.1.1 Hydrologic Model

Determination of the storage required to reduce peak development flows to existing peak flows under specified rainfall inputs requires the use of a hydrologic simulation model of the event type. There are numerous candidate models of this type, but XCG selected the model HEC-HMS, which was developed and is maintained by the U.S. Army Corps of Engineers for the following reasons:

- **i.** It is in the public domain.
- **ii.** It is used widely in Canada and the United States.
- **iii.** It is the successor to HEC-1, the first version of which was published in 1968 and subsequently extensively revised in 1973, 1981 and 1990, and as such has subjected to extensive testing by the hydrologic community.
- **iv.** It incorporated algorithms that have been published and peer reviewed in the technical literature.

2.1.2 Data Requirements

Data required for modelling can be classified as a) meteorological data, b) watershed data or reservoir data.

Meteorological data are essentially rainfall data, which are presented in the form of "Design Storms" and, in the case of Potter Creek, a historical storm that occurred in September, 2004.

Watershed data include physiographic data (drainage area, length and slope), soils data and land use data: all on a sub basin basis. Sub basins are delineated in a process known as "Basin Discretization"; in the case of Potter Creek, QC staff used GIS procedures to discretize the overall basin and determine sub basin data (See Appendix A).

Reservoir data include the locations of all reservoirs in the network and reservoir characteristics for each reservoir.

2.1.3 Three Conditions Modelled

Watershed data must be determined for three watershed conditions:

- **i.** existing (2007) conditions;
- **ii.** post-development conditions, and
- **iii.** post-development conditions with stormwater management measures in place.

2.2 Design Storms and September 2004 Event

2.2.1 Design Storms

Design storms were developed in two steps for a 12-h storm duration for five values of return period: 5, 10, 25, 50 and 100 years.

i. Total rainfall depths were taken from the Atmospheric Environmental Service's updated "Rainfall Intensity – Duration – Frequency Values" for Belleville, Ontario, station number 6150689 (see Table 2.1).

Table 2.1 Design Storm Rainfall Depths*

Return period (years)	5	10	25	50	100
12-hour depth (mm)	52.8	59.0	66.6	72.6	78.1

**** Estimated by Environment Canada***

ii. Hourly values of rainfall depth were determined by applying the AES 12-h southern Ontario temporal distribution (see Table 2.2).

Table 2.2 Design Storm Temporal Distribution

Time Step (h)	1	2	3	4	5	6	7	8	9	10	11	12
Depth (%)	15	25	22	14	12	8	3	1	0	0	0	0

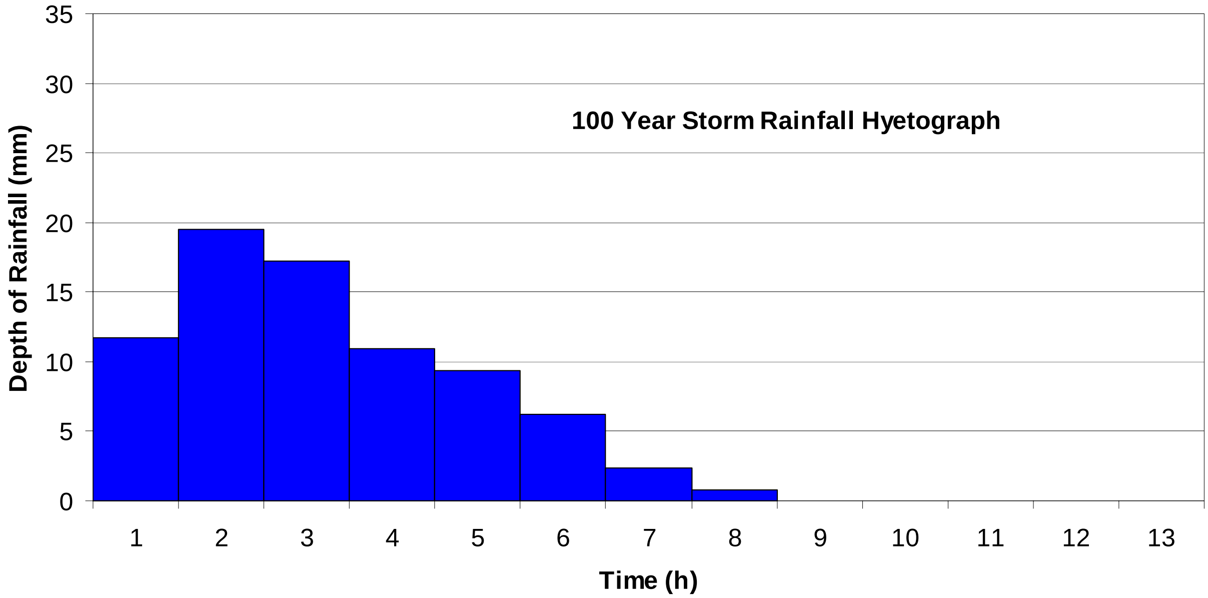
2.2.2 September 2004 Event

A large rainfall event occurred over eastern Ontario on September 9, 2004. Total storm depths recorded at various stations, as given by Klaassen & Seifert 9 (2007) are given in Table 2.3.

Table 2.3 September 2004 Rainfall Event Depths

Location	Belleville	Wilton Cr.	Collins Cr.	Brockville PCC
Depth (mm)	109.2	107.0	137.4	119.8

Inspection of Table 2.3 shows that the event depths exceed the AES estimated Belleville 100-year, 12-hour depth of 78.1 mm (see Table 2.1). In fact, they exceed the AES estimated Belleville 100-year, 24-hour value of 85 mm and the extreme 1-day value of 106.2 mm recorded on July 18, 1921. For comparison purposes, the September 2004 storm recorded at Belleville and the 100-year, AES 12-h design storm are displayed in hyetograph form in Figure 1.



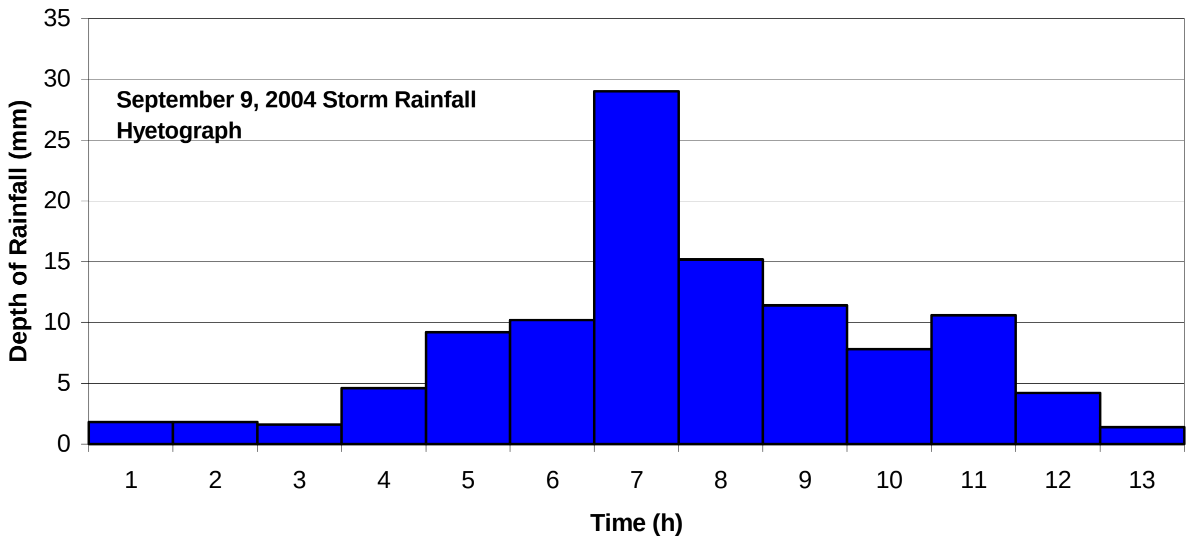


Figure 1 September 2004 Storm and 100-Year

2.3 Basin Discretization

2.3.1 General Principles

The following general principles guided the configuration of the basin into sub basin reservoir, diversion and junction elements 1.

[1] An element refers to a component of the hydrologic event model. Elements incorporate algorithms that attempt to represent flow generation and routing processes.

i. Sub basin elements were provided to represent the watershed routing process for all sub basins obvious on the topographic map.

ii. Sub basin elements were added when a drastic change in land use was anticipated.

iii. Reservoir elements were added when addition of a reservoir was anticipated.

iv. Diversion elements were added when a diversion of runoff into or out of the basin was possible.

v. Junction elements were added to link sub basin hydrographs or tributary hydrographs with each other or with the main branch.

2.3.2 Types of Elements and Parameters

Sub basin elements, (of area A, in km^2) represent two processes: a) abstractions (or losses) from rainfall and/or snowmelt and b) routing of net water input through the sub basin.

In this study, abstractions are generally modelled using the SCS curve number algorithm. This algorithm has only one parameter, the curve number, CN. The values used for CN are those corresponding to antecedent moisture condition II (AMC-II), the average condition preceding annual floods. Values for CN were selected from tables developed by the US Soil Conservation Service, where CN depends on soil type and land use. For post-development conditions, abstractions from directly-connected impervious areas are modelled using the initial abstraction plus continuing loss algorithm, which has two parameters.

Watershed routing is modelled using the SCS dimensionless unit hydrograph algorithm. This algorithm has only one parameter, lag time (or SCS lag). An equation developed by Watt and Chow (1986) was used to estimate lag time using values for sub basin length and slope.

Reservoir elements, which represent detention ponds, are modelled using the modified Muskingum (aka storage indication) method is used to model reservoir routing. This method requires two relations: a) reservoir storage-elevation relation and b) outflow structure hydraulic description.

Diversion elements, which represent diversions, are modelled such that the diverted flow is a specified portion of the inflow; with the proportion able to vary with the inflow.

Junction elements are simply addition elements. The outflow from a junction is equal to the sum of all inflows to the junction plus any flow diverted in and less any flows diverted out.

2.4 Existing Conditions

2.4.1 Overall Basin

General description: The Potter Creek drainage basin (see Figure 2) extends from near Tucker's Corners in the north to the Bay of Quinte in the south. Its drainage area at Highway 2 is approximately 30 km^2, and includes land in the City of Belleville and Quinte West.

Drainage network: Potter Creek has seven tributaries with varying drainage area and land use. Previous study reports contained the names tributary 1, tributary 2 and tributary 3 for the three south-eastern tributaries with the number increasing in a counter clockwise direction. To avoid confusion, we retain this naming convention, and continue so that the three tributaries originating north of highway 401 are termed tributaries 4, 5 and 6, and the remaining tributaries 7 and 8 (see Figure 2).

Soils: The distribution of soil types is shown in Figure 3. Data for this figure were taken from the ***Soil Survey of Hastings County*** (Gillespie et al. 1962).

Sandy loams and muck are found along the northern edge of the watershed towards Vermilyea Road. Highway 401 effectively divides the watershed into north and south sections with clay and clayey loams found in the middle vicinity around the 401 area. Much further south of the 401, between Highway 2 and Moira Street, the predominant soil type is loam with some clay and clay loam deposits. Small areas of sandy loam are noted in this section as well.

Developed areas (existing conditions): Developed areas (under existing conditions) are generally restricted to the southeast corner of the drainage basin (see Figure 2). Large impervious areas elsewhere in the basin include strip development along highway 401, Walbridge Road and the Loyalist College development.

Residential development is evident along the eastern portion of the Potter Creek watershed, the majority of which are medium density single-family homes. Well-established residential areas are found north of Moira Street and west of Sidney Street. South of Moira Street is a mixture of residential and industrial properties with the watershed boundary located along the industrial area.

Commercial development is also evident along the 401-corridor area as well. Bellevue Road runs adjacent to the north side of the 401 and residential, commercial, agricultural and vacant lands are located along this secondary road. To the south of the 401 is Bell Boulevard, which currently is the location for a few commercial enterprises as well as vacant land.

2.4.2 Sub-basins

Sub basins were defined by first placing junction nodes at junctions of tributaries with other tributaries and with the main channel (see Figure 4).

Sub basin soils for existing conditions; junctions and sub basin boundaries are shown in Figure 5.

Sub basin parameters for existing conditions (area, time to peak and curve number) were determined by GIS procedures (Appendix A) and are listed in Table 2.4.

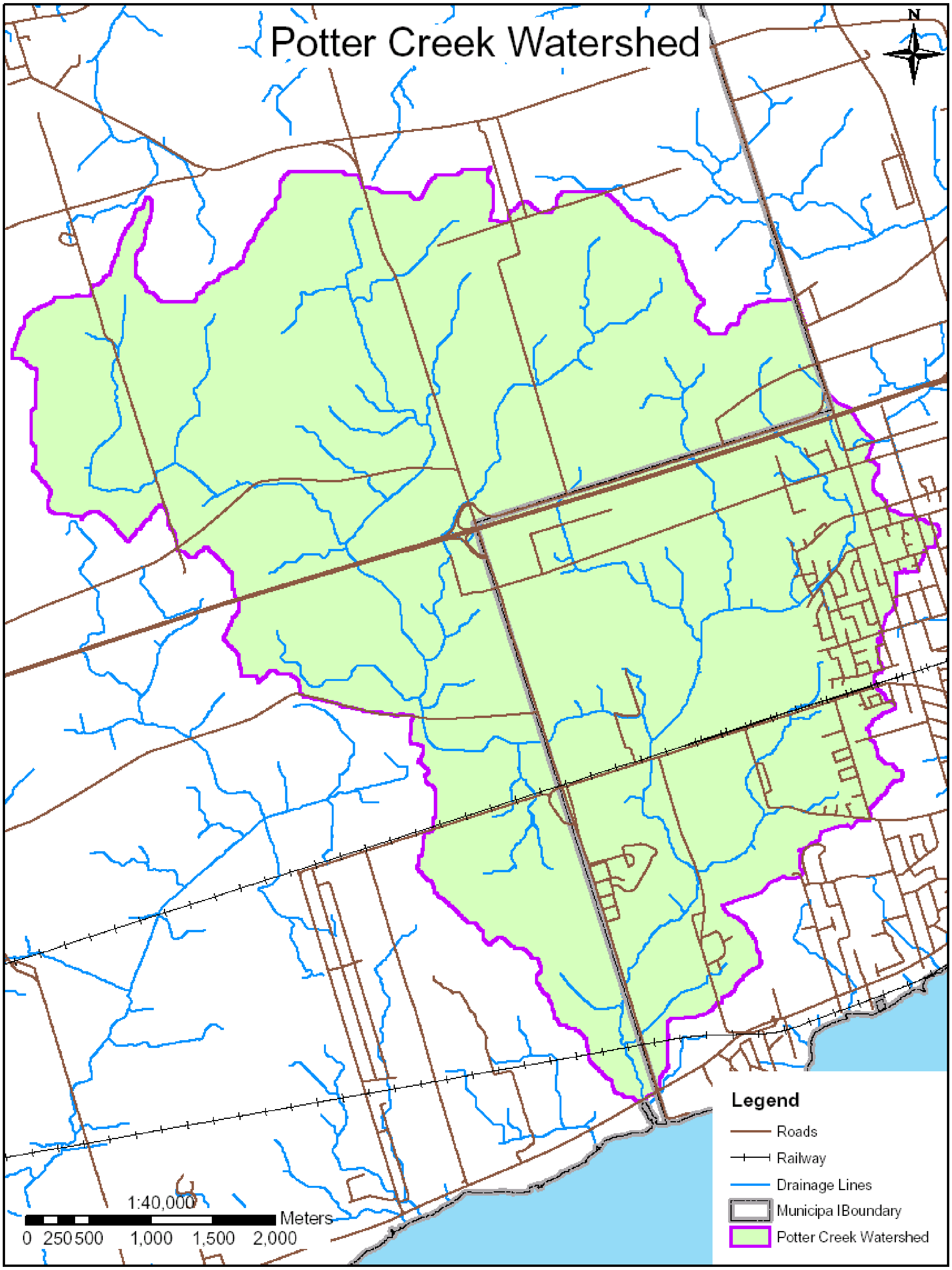


Figure 2 Potter Creek Drainage Basin

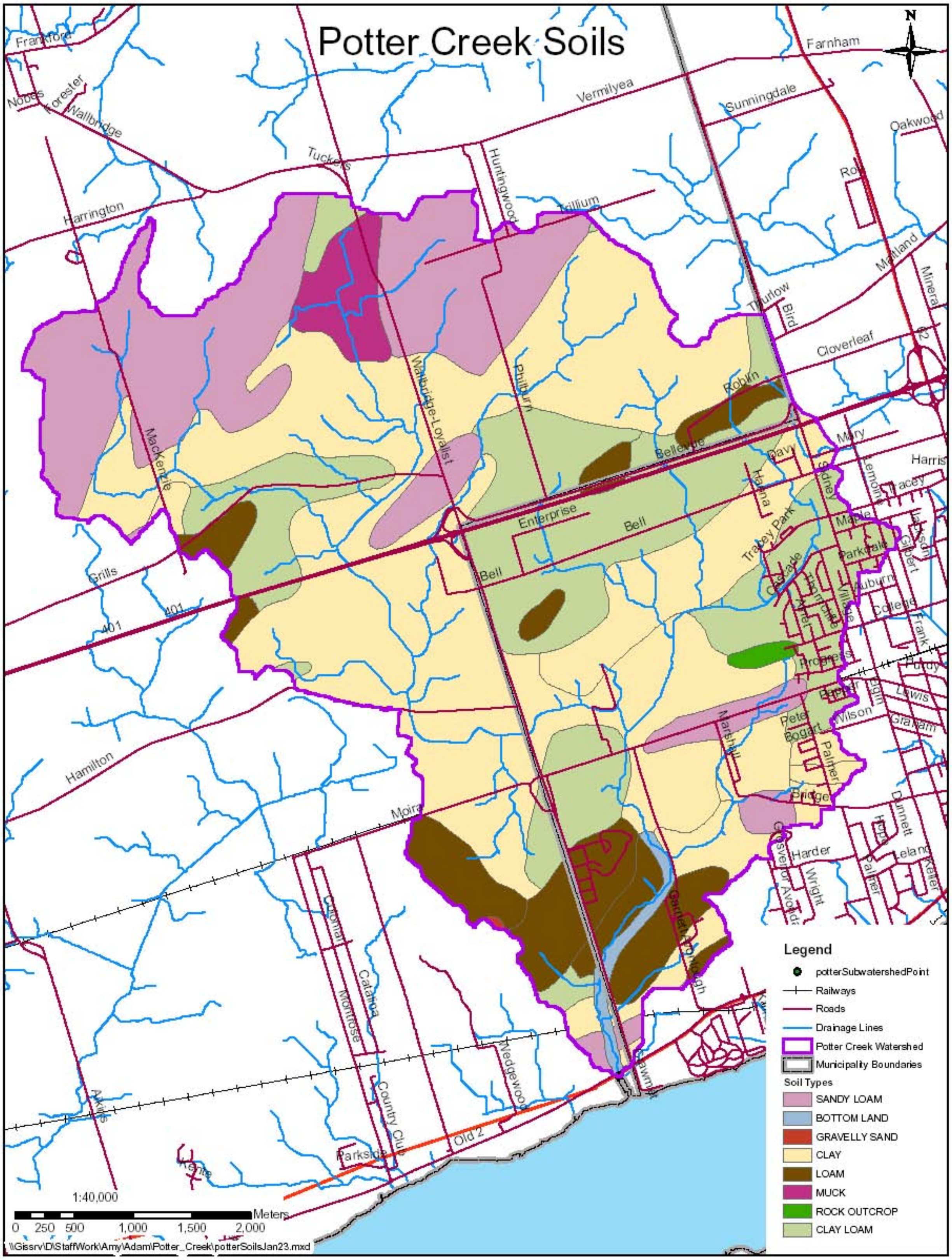


Figure 3 Potter Creek Soil Types

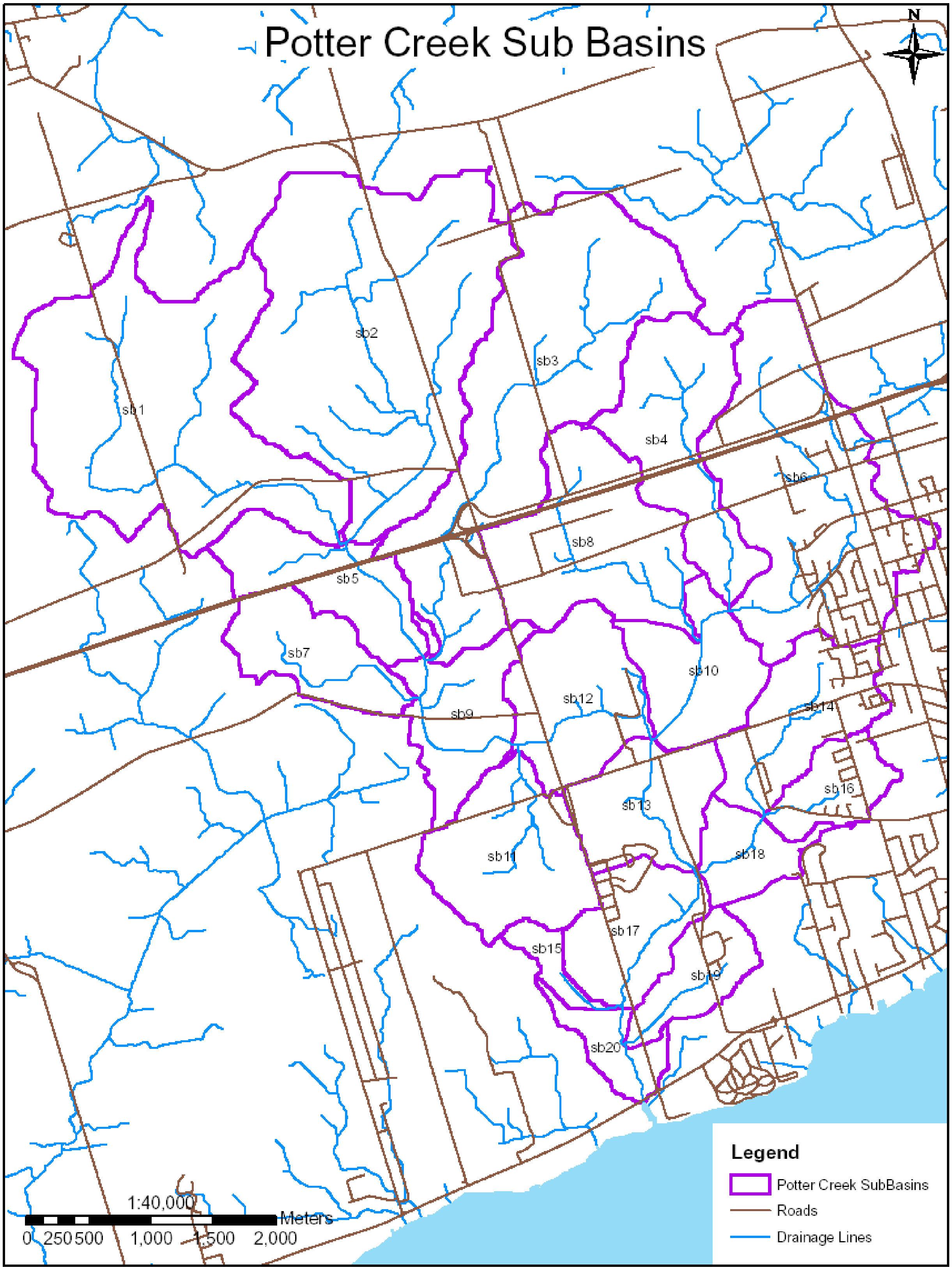


Figure 4 Potter Creek Sub basins

Table 2.4 Sub basin Parameters - Existing Conditions

Sub basin	Area (km^2)	Time to Peak (h)	SCS Curve Number
1	3.89	0.80	67
2	4.55	1.10	70
3	3.63	1.67	75
4	1.51	0.97	81
5	1.02	0.74	81
6	3.00	1.12	81
7	0.86	1.09	83
8	1.82	0.84	80
9	0.97	0.97	84
10	0.98	1.46	80
11	1.36	0.65	78
12	1.23	0.87	81
13	0.99	0.87	78
14	1.10	1.28	80
15	0.21	0.37	69
16	0.52	0.57	79
17	0.93	0.75	66
18	0.51	0.45	80
19	0.57	0.54	73
20	0.39	0.66	66

The existing developed areas in sub basins sb6, sb14, sb16 and sb19 are connected to the Belleville storm sewer system. The model for each of these areas includes a diversion element whereby all flows up to a certain “diversion” value, which corresponds to the capacity of the storm sewer for the area, is diverted out of the model for Potter Creek. For flows in excess of this value, the difference between the flow and the diverted flow drains to the outlet of the sub basin. Diversion values for sub basins sb6, sb14, sb16 and sb19 were taken as 1.2, 0.73, 0.47 and 0.14 m^3/s respectively.

These values are based on the capacity of the respective storm sewers, operating without obstruction and flowing “full”. They represent the maximum diverted flow possible, which may or may not be the case under 100-year conditions. In the final design of the storage facilities for these sub basins, two items should be checked: the capacity of the major system, and the potential for reduced capacity of the minor system due to either blockage or hydraulic constraints imposed by major system flows.

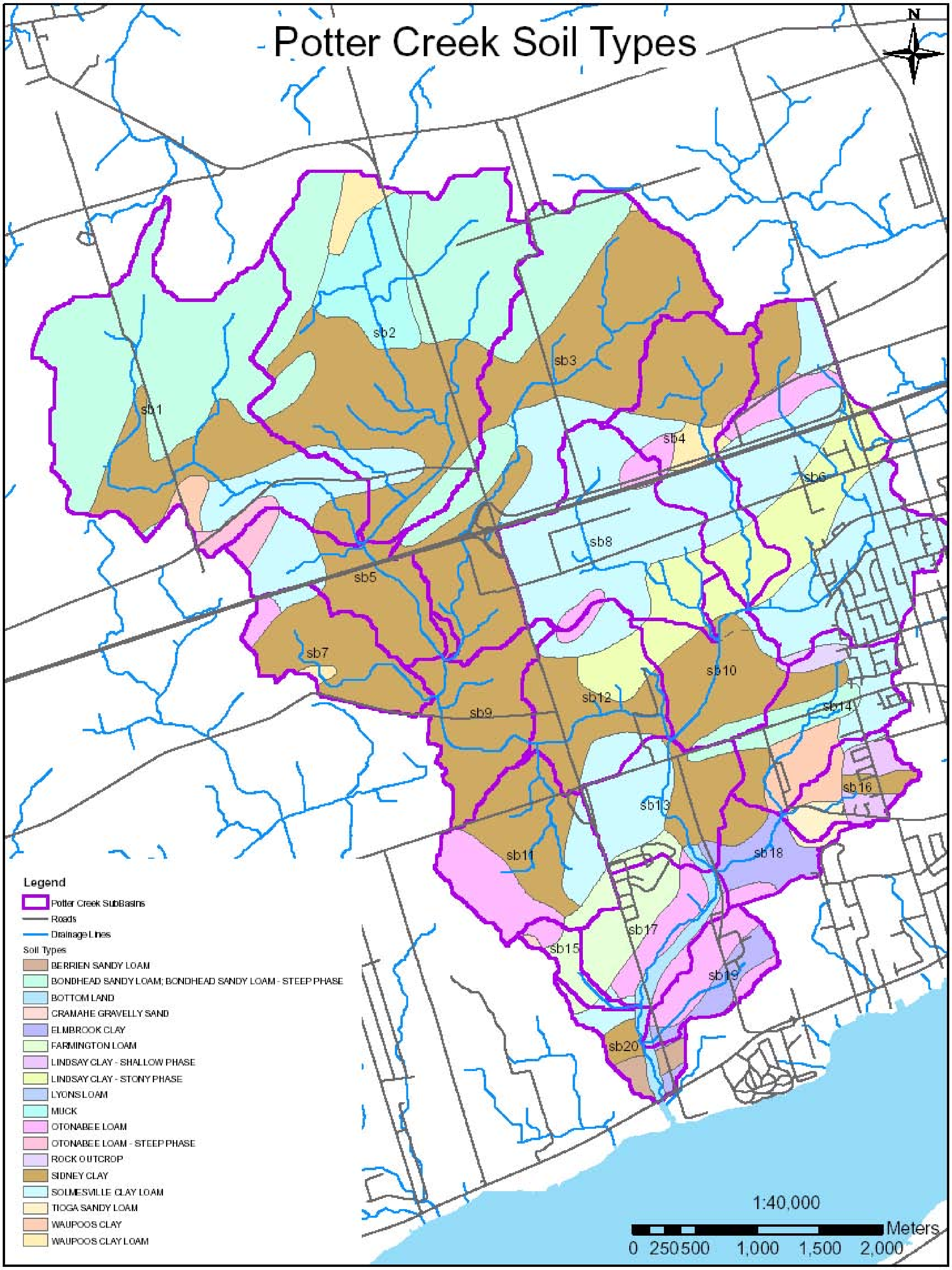


Figure 5 Potter Creek Discretization and Soils

Sub basin flows – existing conditions: The peak flow at the outlet of each sub basin from the HEC-HMS model for the case of the 12-hour, 100-year rainfall input and with the parameter values listed in Table 2.4, is given in Table 2.5. **These flow values should be considered benchmarks against which flows for developed conditions are compared**.

The unit peak flows (in m^3/s/ha) are also tabulated. They range from 0.01 to 0.02 m^3/s/ha – the lower values reflecting lower SCS curve numbers (e.g. sub basin 1) and diversions (e.g. sub basins 6, 14, 16 and 19) and/or longer times to peak (e.g. sub basin 3). The unit peak flow at the outlet, at Highway 2, is calculated to be 36.2/3010 = 0.012 m^3/s/ha = 1.3 $m^3/s/km^2$.

Table 2.5 Sub basin and Potter Creek Flows - Existing Conditions

Sub basin	Sub basin Peak Flow (m^3/s)	Sub basin Unit Peak Flow (m^3/s/ha)	Location on Main Branch	Peak Flow [1] (m^3/s)
1	3.5	0.009	401 1km west of WL	8.3
2	4.8	0.011	Hamilton St.	16.0
3	4.8	0.013	WL north of Moira	19.2
4	2.6	0.017	Moira St./CNR	31.6
5	2.0	0.020	DS of Trib 2 (Avonlough)	34.7
6	4.3	0.014	CPR	36.1
7	1.7	0.020	Highway 2	36.2
8	3.0	0.017		
9	2.1	0.022		
10	1.5	0.015		
11	2.3	0.017		
12	2.6	0.021		
13	1.5	0.015		
14	1.3	0.011		
15	0.2	0.010		
16	0.4	0.008		
17	0.8	0.009		
18	1.1	0.022		
19	0.5	0.010		
20	0.4	0.010		

[1] To determine these flows on the main branch, channel elements were added to represent the delays between the sub basin outlets and the downstream end of the basin at highway 2. The linear channel lag algorithm was used.

2.4.3 Comparison of Flows from 100-year Rainfall and 2004 Event

The 2004 event rainfall (see Figure 1) was applied to the same basin model used to determine flows under existing conditions (see Table 2.4), but with one change. Instead of using SCS curve numbers for antecedent moisture condition II, the average condition preceding annual floods, XCG used curve numbers for antecedent condition I where soils are dry, but not to the wilting point. The Environment Canada records for Belleville show no rainfall for a week preceding the 2004 event and only 38.8 mm for the month of August, 2004, which included only one day with rainfall exceeding 10 mm (August 27). The Soil Conservation Service limit for AMC I is a total 5-day antecedent rainfall of less than 0.5 inches during the dormant season and less than 1.4 inches during the growing season. Clearly, the 5-day antecedent precipitation of zero for the 2004 event is consistent with AMC I.

The hydrographs at the basin outlet for the 2004 event (with AMC I) and the AES 100-year rain (AMC II) are compared in Figure 6. Clearly, both volume of runoff and peak flow are larger for the AES 100-year rain even though the 100-year input rainfall is smaller than the 2004 event rainfall.

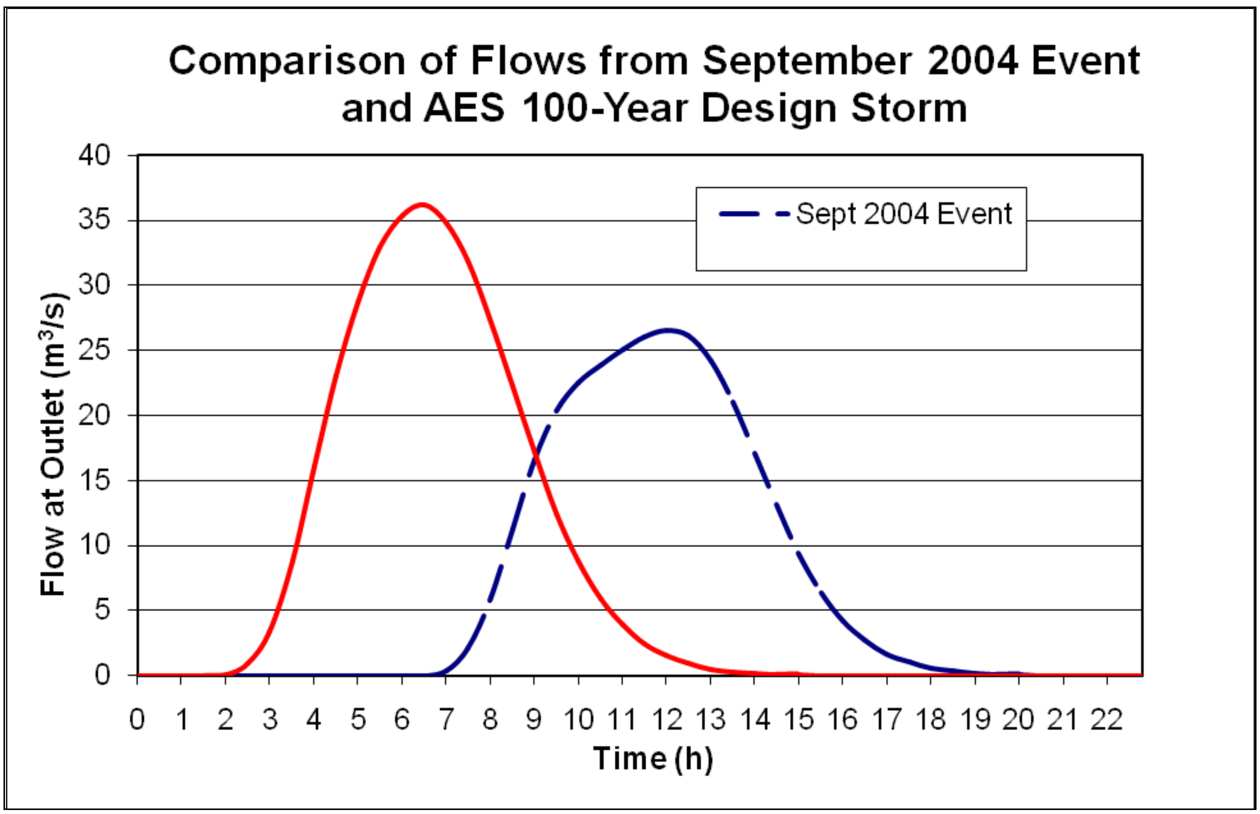


Figure 6 Hydrographs for 100-year Rainfall and 2004 Event

Quinte Conservation staff reported that the maximum water level upstream of the culvert at highway 2 during the September 2004 event was well below the flood line, which is based on a flow of 39.5 m3/s. This observation is consistent with a lower peak flow (i.e. 27 m3/s vs. 36 m3/s). Environment Canada records for Lake Ontario for September 9, 2004 show water surface elevations of 74.72 m and 74.71 m at Cobourg and Kingston respectively; these values are about 1/3 m lower than the value used as the starting value for the hydraulic model used for floodline determination.

2.5 Post Development Conditions

2.5.1 Planned Development

Planned development was defined by the official plans of the City of Belleville and the City of Quinte West. This information is shown on Figure 7. In summary, there is no drastic change in land use anticipated for the following areas:

- **i.** the area north of highway 401;
- **ii.** the area west of Wallbridge Road; and
- **iii.** the existing developed lands in the southeast part of the basin.

For the remainder of the basin, the official plans designate development and future land use that provides a significantly different land use than currently designated. A large commercial area just south of the 401 corridor is centred along Bell Boulevard. A smaller commercial area is designated along Moira Street. These areas would show the largest changes in runoff due to the high imperviousness of the commercial structures and associated parking facilities.

The remaining development area is designated residential with varying densities. The majority of these residential development areas are designated low-density housing. Green areas are evident in the proposed developments and they focus around Potter Creek and its associated tributaries.

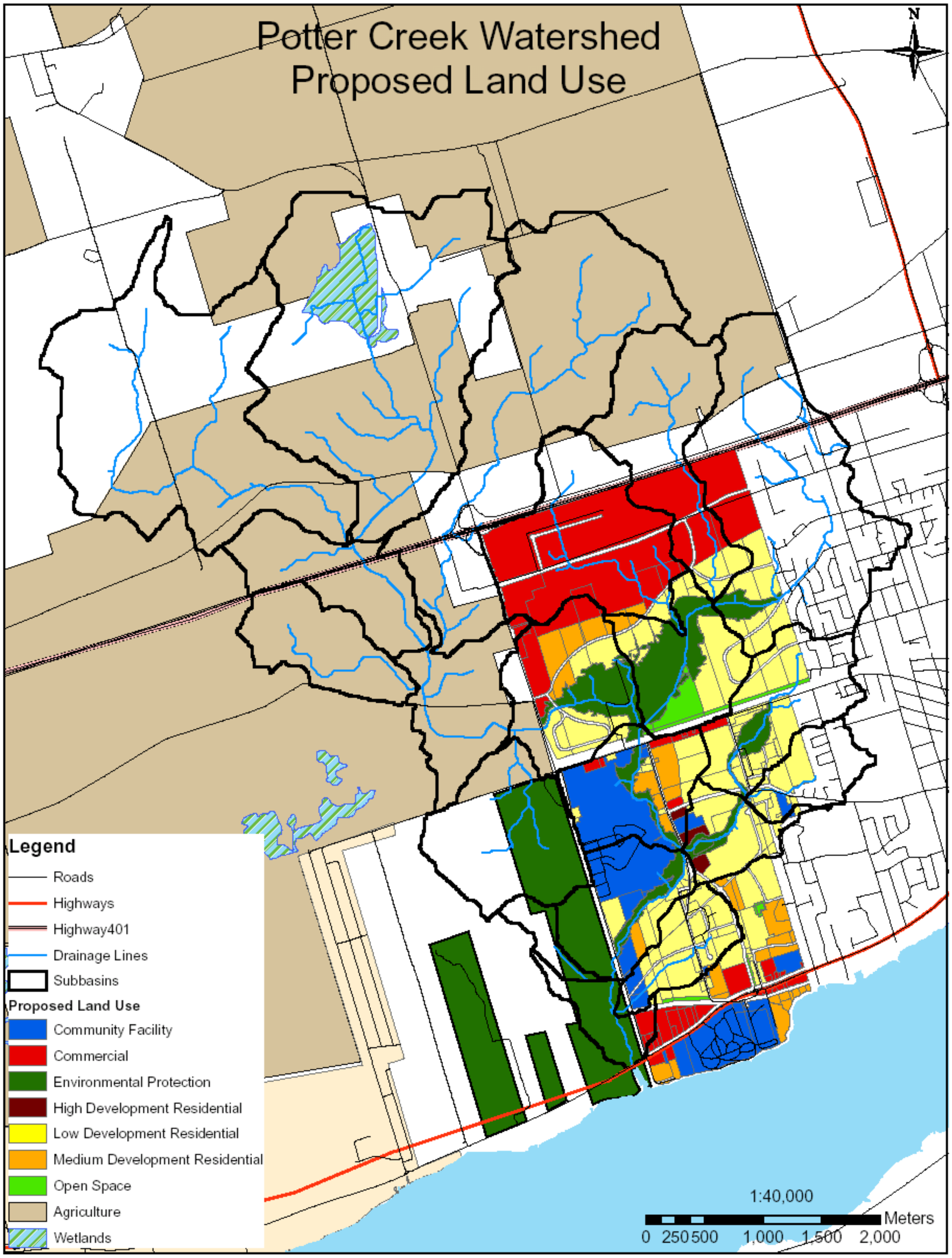


Figure 7 Potter Creek – Planned Development

2.5.2 Sub-basin Parameters and Flows

Sub basin parameters for developed conditions (see Table 2.6) were determined as follows.

i. Sub basins were divided into two categories: a) those for which a change in land use is planned according to the official plans, and b) those for which no change was planned.

ii. For each category a) sub basin, the developed area and the impervious and directly-connected impervious portions were determined according to the portion of the area to be developed and the type of planned development (see Figure 7).

iii. Each category a) sub basin is represented by two parallel watershed elements: one representing the directly-connected impervious areas and one representing the remaining sub basin area.

iv. Abstractions from the directly-connected impervious portions are modelled as an initial abstraction of 2 mm and zero continuing abstraction.

v. Abstractions from the remaining area, which includes the undeveloped portion, the pervious areas of the developed portion and the non-directly connected areas of the developed portions are modelled using the SCS curve number algorithm. The curve number for each sub basin is calculated in two steps. First, an <u>equivalent</u> curve number is calculated for the pervious and non-directly connected impervious parts of the developed portion. Second, a <u>weighted</u> curve number is calculated to include the undeveloped portion.

vi. Times to peak for the directly-connected impervious portions are taken as 75% of the time to peak for existing conditions.

vii. Times to peak for the remaining portions are taken as the same as for existing conditions.

Table 2.6 Sub basin Parameters – Post-development Conditions

Sub basin	SCS Curve Number	Impervious Level (%)	Directly Connected Impervious Level (%)
1	67	-	-
2	70	-	-
3	75	-	-
4	81	21	11
5	81	-	-
6	81	40	22
7	83	-	-
8	80	61	33
9	84	-	-
10	80	16	9.0
11	78	-	-
12	81	35	20
13	78	12	7.0
14	80	49	28
15	69	-	-
16	79	49	28
17	66	18	10
18	80	43	24
19	73	41	23
20	66	-	-

Note: Impervious level and directly-connected impervious level are required for only those sub basins where a change in land use is planned.

Sub basin flows - developed conditions: The output from the HEC-HMS model for the case of the 12-hour, 100-year rainfall input and with the parameter values listed in Table 2.6 is given in Table 2.7.

Table 2.7 Sub basin – Post - development and Existing Conditions

Sub basin	Post development Peak Flow[1] (m^3/s)	Existing Peak Flow (m^3/s)
1	3.5	3.5
2	4.8	4.8
3	4.8	4.8
4	**3.7**	2.6
5	2.0	2.0
6	**9.7**	4.3
7	1.7	1.7
8	**7.2**	3.0
9	2.1	2.1
10	**2.1**	1.5
11	2.3	2.3
12	**4.0**	2.6
13	**1.9**	1.5
14	**3.9**	1.3
15	0.2	0.2
16	**2.0**	0.4
17	**1.2**	0.8
18	**2.0**	1.1
19	**1.6**	0.5
20	0.4	0.4

[1]Flow values that have changed from existing conditions are bold.

2.5.3 Comparison – Developed and Existing Conditions

Inspection of Tables 2.5 and 2.7 reveals the following.

- **i.** There is no change for sub basins 1, 2, 3, 5, 7, 9, 11, 15, and 20.
- **ii.** There is a modest increase in peak flow (up to a factor of 1.5) for sub basins 4, 10, 12, 13 and 17.
- **iii.** There is a larger increase in peak flow for sub basins 6, 8, 14, 16 and 18.

3. POTTER CREEK STORMWATER CONTROL

3.1 General

3.1.1 Stormwater Management

> *Urban stormwater management is the conceptualization, planning, design, construction, and maintenance of stormwater control facilities in urban/ urbanizing drainage basins, and includes all related political, social, and economic considerations. While this definition does not necessarily involve new construction, it includes such facilities as open channels, curbs and gutters, storm sewers, detention/retention ponds and associated structures, water quality enhancement measures, special structures (energy dissipators, transitions, inlets, etc.) and others.*
>
> *(WEF ASCE 1992)*

In its earlier form, stormwater management focused, almost exclusively, on mitigating the negative impacts of downstream flooding and erosion through partial replacement of the natural storage lost by urbanization by constructed storage: a) at a lot level by roof storage or b) at the end of the pipe with stormwater pond storage or with subsurface storage. Today, this aspect of stormwater management is sometimes referred to as "quantity control" as distinguished from "quality control", which is realized by the provision of stormwater quality best management practices (i.e. BMPs).

3.1.2 Major-Minor System Approach

The major-minor system approach is now accepted by many jurisdictions throughout North America and is included in drainage manuals/guidelines for many Ontario municipalities. WEF ASCE (1992) provide a detailed description, but essentially,

i. the **minor** drainage system includes roof gutters, streets, stormwater inlets, storm sewers, open channels and street culverts. It is designed to convey (without surcharging) the peak discharge from more frequent storms up to the design frequency of the system (i.e., the 2 or 5 or 10-year storm, etc);

ii. the **major** system includes natural streams and valleys and constructed structures such as streets, swales, channels and ponds. It is designed to convey, in parallel with the minor system, the discharge from less frequent storm events such as the 100-year storm or regional storm.

3.1.3 Stormwater Management BMPs

Discharges of urban stormwater into receiving waters result in negative impacts other than flooding and erosion. These impacts include pollution, ecosystem degradation and impairment of beneficial water uses (WEF ASCE 1998). To prevent or mitigate such impacts, stormwater management has been enhanced and implemented through stormwater management best management practices or BMPs.

In Ontario, the Ministry of the Environment initiated the development of a **Stormwater Management Practices Planning and Design Manual** (MOEE 1994). This document focused on water quality. The 1994 manual was updated in 2003 (MOE 2003) to include updated and expanded topics. Citations in this chapter refer to the 2003 version.

3.1.4 Master Drainage Planning

WEF ASCE (1992) state:

> *Master planning is one of the most widely used and frequently misunderstood terms in drainage practice. There are few published definitions, but a master plan typically addresses such subjects as characterization of site development, grading plan, peak rates of runoff and volumes for various return frequencies, locations, criteria and sizes of detention ponds and conveyances, measures to enhance runoff quality, pertinent regulations and how the plan addresses them, and consistency with secondary objectives such as public retention, aesthetics, protection of public safety, and groundwater recharge.*
>
> *In its simplest form, a master plan may only identify the essential elements, alignments, and functions of a drainage system. Even at this conceptual level, the master plan should be based upon estimates of peak and total discharges for some selected runoff recurrence interval(s), in turn, should be selected based on local standards and risk assessment, as discussed earlier in this chapter.*
>
> *The next level of master planning should establish specific criteria consistent with acceptable risk, including design discharges and water surface profiles and elevations. Head losses at waterway crossings and other constructions or obstructions should be recognized in development of the water surface profiles. This level of master planning defines the ultimate drainage system components desired and provides information for their preliminary design and cost estimation.*

The above description of levels of master drainage planning applies to a wide range of jurisdictions and rate of development ranging from the rapid development case where an entire drainage basin is fully developed over a period spanning less than a year to a fairly slow rate of development where the development period is of the order of a decade. In the former case, a high level of master planning is possible and efficient. In the latter case, however, the lack of development plans for the entire basin at the time of initial development precludes detailed specifications of locations and designs of drainage structures.

In an ideal world, drainage planning and land use planning would be conducted in parallel and on an iterative basis, with the “first stage” drainage planning (i.e. identification of stream corridors and location of storage structures) providing one input into the first stage of the official plan, the “second stage” drainage planning (e.g. sizing of storage structures) providing input into a draft official plan, etc.

Unfortunately, ours is not an ideal world and in many cases development leads both land use planning and master drainage planning and, as a result, a less than perfect developed drainage system results. For example, it is generally not feasible to determine size and location of individual storm water management facilities in advance of actual development plans, even though the official plan for the area may be known. Development of a sub basin may occur over a period of years and in this case more than one facility may be the preferred solution for both the developer and municipality. In other cases, where the entire sub basin is developed by a single developer over a shorter time period, a single larger facility maybe preferred.

3.2 Potter Creek Stormwater Quantity Control

3.2.1 Design of Major System

The major-minor system approach was briefly described in section 3.1. The City of Belleville has design guidelines for the design of the minor system, but no explicit requirement for the design of the major system, a common state of affairs for many Ontario municipalities. However, recent experience with major storms in Ontario and elsewhere indicates that the major system failed to convey the storm discharge with concomitant damages resulting from flooding and short circuiting to the sanitary drainage system. Accordingly, XCG recommends that explicit design of the major system be a requirement for all stormwater management plans within the Potter Creek drainage basin.

WEF ASCE (1992) provide guidelines on “street and intersection design” under various headings including “Street Capacity for the Major System design Runoff” and “Intersections” as well guidelines on “Major Drainageways (Open Channels)”. These or similar guidelines should be followed to ensure that

- **i.** major system runoff follows the intended major system,
- **ii.** there are no short circuits to the sanitary system,
- **iii.** sub basin flows are directed to the intended facility and do not bypass it, and
- **iv.** pond outflows are directed to the intended receiving water.

3.2.2 Storage Requirements for Quantity Control

Approximate locations of required storages correspond to the outlets of sub basins 4, 6, 8, 10, 12, 13, 14, 16, 17, 18 and 19 (See Figure 8).

Magnitudes of required storages (water quantity considerations): These values were determined by adding a storage element to the hydrologic model for a particular sub basin and adjusting the storage until post-development peak flows were equal to existing peak flows. Values listed are for the case of the 100-year rainfall input – the governing case.

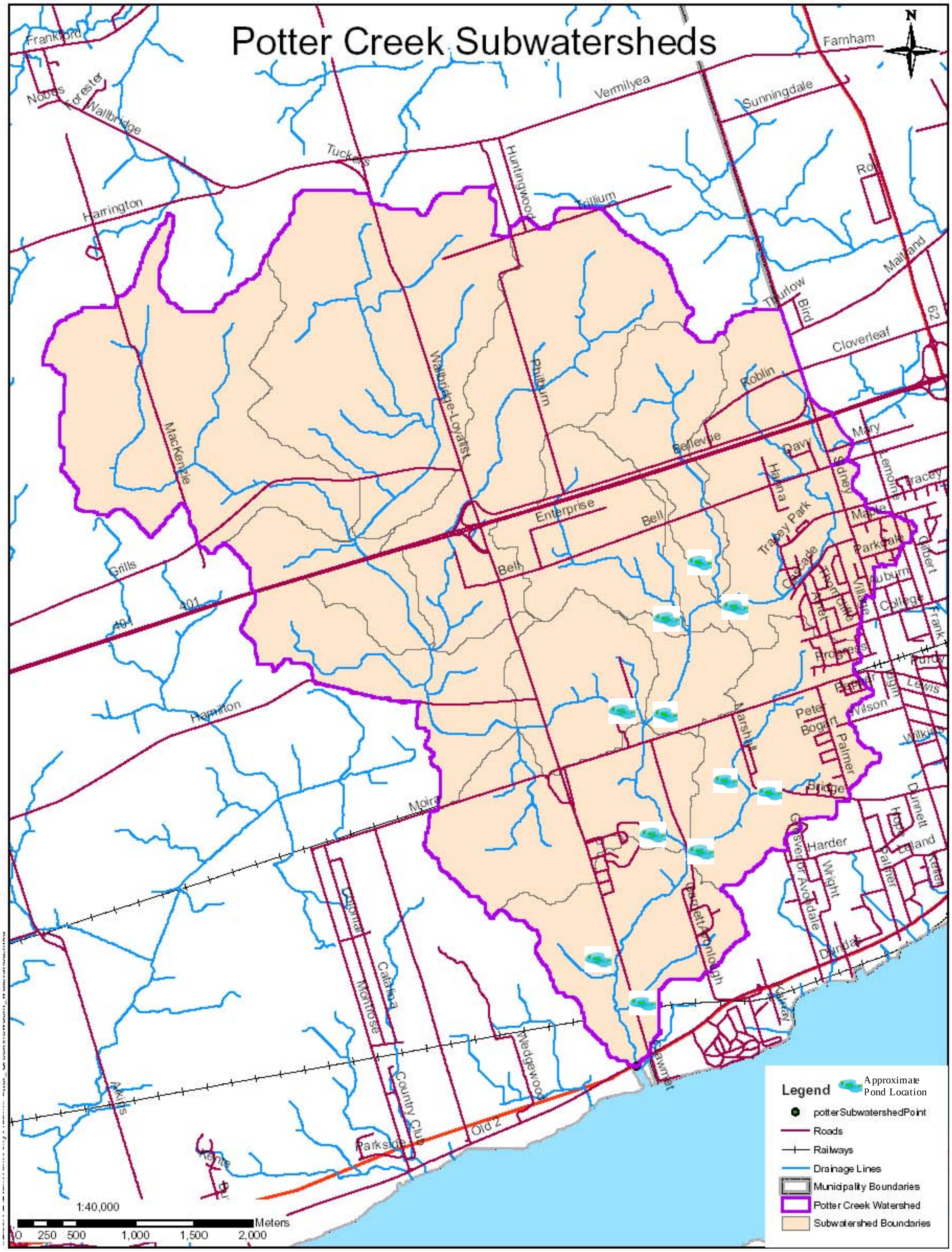


Figure 8 Approximate Locations of Required Storage

3.2.3 Comparison of Peak Flows

Table 3.1 provides a) the approximate storage required to reduce post-development peak flows to existing conditions peak flows, and b) a comparison of peak flows from the 100-year rainfall input for three conditions:

i. existing (2007) conditions;

ii. post-development conditions; and

iii. post-development conditions with stormwater management measures in place at or near sub basin outlets.

Table 3.1 Storage and Comparison of Peak Flows* for Three Conditions

Sub basin	Storage (1000 m^3)	Peak Flow for Existing Conditions (m^3/s)	Peak Flow for Post-development (m^3/s)	Ratio of Post Development to Existing Flows	Peak Flow for Controlled Post-development (m^3/s)
4	32.3	2.6	3.7	1.4	2.6
6	86.7	4.3	9.7	2.3	4.3
8	79.8	3.0	7.2	2.4	3.0
10	20.0	1.5	2.1	1.4	1.5
12	34.0	2.6	4.0	1.5	2.6
13	15.8	1.5	1.9	1.3	1.5
14	34.6	1.3	3.9	3.0	1.3
16	15.5	0.4	2.0	5.0	0.4
17	12.1	0.8	1.2	1.5	0.8
18	15.8	1.1	2.0	2.9	1.1
19	16.4	0.5	1.6	3.2	0.5

****resulting from 100-year, 12-h rainfall input***

Inspection of Table 3.1 shows that,

i. for all sub basins, the peak flow for controlled post-development conditions is equal to the peak flow for existing conditions,

ii. the ratio of post-development peak flow to existing conditions peak flow ranges from 1.3 (sub basin 13) to 5.0 (sub basin 16), reflecting the level of development.

3.3 Potter Creek Stormwater Quality Control

3.3.1 Overview

In this document, stormwater quality control is considered in two categories: a) control at sub basin outlets (or end-of-pipe control) by way of **central water quality facilities** in the form of off-line extended stormwater detention ponds and b) control throughout the Potter Creek watershed through **source control measures**, which include a range of practices and facilities.

3.3.2 Central Water Quality Facilities

Approximate locations of extended detention ponds: The extended detention ponds are at the outlets of the sub basins listed in Table 3.1 and shown in Figure 3. In most cases, the extended detention pond will be sized and designed so as to provide the storage required for quantity control and storage required for quality control.

Magnitudes of required storage (water quality considerations): The magnitudes of the required storage are given in Table 3.2. These storages were determined by applying provincial water quality sizing criteria (MOE 2003), wherein required storage volume is a function of "Protection Level" and "Impervious Level". The required storage volume (in m^3/ha) includes an extended detention storage of 40 m^3/ha, the remainder representing permanent pool storage.

Table 3.2 Required Storage Volume for Quality Control

Sub basin	Area (ha)	Impervious Level (%)	Unit Storage[1] (m^3/ha)	Total Quality Storage (1000 m^3)	Extended Detention Storage[2] (1000 m^3)	Permanent Pool Storage (1000 m^3)
4	151	21	104	15.7	6.0	9.7
6	300	40	152	45.5	12.0	33.5
8	182	61	206	37.5	7.3	30.2
10	98	16	93	9.1	3.9	5.2
12	123	35	140	17.3	4.9	12.3
13	99	12	82	8.2	4.0	4.2
14	110	49	176	19.3	4.4	14.9
16	52	49	174	9.1	2.1	7.0
17	93	18	99	9.2	3.7	5.4
18	51	43	159	8.1	2.0	6.1
19	57	41	144	8.8	2.3	6.5

[1] From MOE (2003) table 3.2 for Enhanced Protection Level.
[2] 40 m^3/ha as specified by MOE (2003)

Magnitudes of required storages for erosion control: MOE (2003) also includes guidelines for erosion control. Comparison of Figures C.1 and C.2 in Appendix C (in the MOE document) with Table 3.2 above indicates that for typical impervious levels and directly connected impervious levels, the volume required for quality control will govern.

3.3.3 Source Control Measures

In this report, source control measures are classified in the same manner as in an excellent review article by Marsalek (2001), that is, a) source control measures reducing stormwater quantity, and b) source controls enhancing water quality.

> *Source control measures reducing stormwater quantity are generally those which reduce the extent of impervious areas; divert runoff from impervious areas onto pervious areas; enhance hydrologic abstractions on natural or man-made surfaces by detention/retention, infiltration and evaporation; and, reduce stormwater flows by storage and reuse. While these measures primarily control runoff quantity, they also*

improve stormwater quality by immobilising stormwater pollutants on catchment surfaces, or diverting them to soils, or groundwater. A brief overview of selected measures follows.

Source controls enhancing stormwater quality are generally policies and related measures which reduce or eliminate entry of numerous pollutants into stormwater. These measures are generally designed to promote the prevention of stormwater pollution by various activities conducted by the public, municipalities and small businesses. Many such measures are described in great detail in the literature and are recommended for application in both existing and new urban developments.

Source control measures reducing stormwater quantity: Marsalek classifies these as:

- **i.** land use planning and management practices (e.g. buffers for streams and wetlands), and
- **ii.** lot-level source controls (reduction of directly connected impervious areas, parking lot and roof top storage, runoff conveyance by grassed swales and through filter strips, runoff storage and reuse, etc.).

Source controls enhancing water quality: Marsalek classifies these as:

- **i.** public education, awareness and participation;
- **ii.** modified use, releases and disposal of chemicals entering stormwater (e.g. household chemicals and hazardous wastes, garden chemicals and road salts);
- **iii.** enforcement of sewer ordinances (e.g. illegal dumping, illicit connections to storm drains);
- **iv.** housekeeping practices(e.g. storage of materials that could end up in stormwater, vehicle spill controls, vegetation controls);
- **v.** reduction of stormwater pollution by construction activities (i.e. erosion control, sediment collection, site water control, equipment storage and maintenance, materials storage and litter control); and
- **vi.** maintenance activities (e.g. street cleaning, maintenance of parks and other public places, domestic recycling and waste collection).

Detailed discussion of these measures is contained in MOE (2003). Selection of which measures are appropriate for Potter Creek depends on physical, political and economic considerations which are best determined by the appropriate departments (Planning, Parks, etc.) in the City of Belleville and Quinte West. However, in our opinion, two key measures are essential: minimization of directly-connected impervious area and sediment control.

3.4 Potter Creek Stormwater Control Facilities

3.4.1 Overview

The approximate location and volumes of required storages are given in Figure 8 and Table 3.1 respectively. In each case, the storage will be provided in the form of a stormwater control facility, specifically an extended stormwater detention pond.

i. The values of storage tabulated do not necessarily have to be provided in one reservoir at one location, but could be distributed in more than one reservoir for a variety of reasons (e.g. site conditions, site restrictions on maximum size, staged development etc.). In the case of one or more reservoirs, the sum of the individual storage values for each individual facility could exceed the tabulated values, but cannot be smaller.

ii. All ponds will be off-line, that is, the outflow from the pond (see Figure 9) will be directed to the existing watercourse (see Figure 8).

Although final designs for these facilities are beyond the scope of this report, the required storage volume, a generic design and design guidelines can be provided. The generic design is common to many North American jurisdictions, as are the design guidelines.

3.4.2 Components and Design Guidance

An extended detention pond includes the following components:

i. inlet,

ii. sediment forebay,

iii. pond,

iv. outlet structure, and

v. surrounding buffer area

as illustrated in Figure 9.

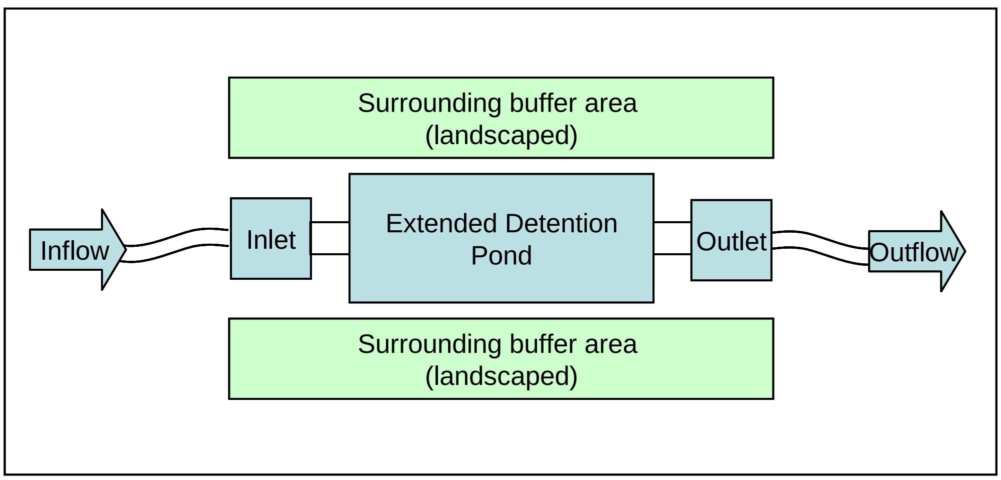


Figure 9 Schematic of an Extended Detention Pond

MOE (2003) provides design guidance for the design of wet ponds for water quality objectives for various "design elements" that include the components listed above as well as the area surrounding the pond including provision for maintenance access. An abbreviated form of this guidance is provided in Table 3.3. Also shown in Table 3.3 are those portions of the City of Belleville Guidelines (see Appendix B) that deal with pond dimensions.

Table 3.3 MOE and City of Belleville Design Guidance for Stormwater Ponds

Element	MOE Minimum Criteria	MOE Preferred Criteria	Belleville Guidelines
Minimum Drainage Area (ha)	5	10	-
Storage Volume (m^3/ha)	MOE Table 3.2	Increase by expected ice volume Increase active storage to 25 % of total volume.	-
Detention Time	24 hr	24 hr	-
Forebay: minimum depth maximum area maximum volume	1 m 33% of total permanent pool	1.5 m 20 % of total permanent pool	Permanent pool: 0.9-1.2m <34% of total pond surface area
Minimum length: width	3:1 overall 2:1 for forebay	4:1 to 5:1	-
Permanent Pool: maximum depth mean depth	3 m 2.5 m	2.5 m 1 to 2 m	0.9-1.2 m
Active Storage Depth: water quality control total including quantity	1.5 m 2 m	1 m 2 m	Max. depth = 1.3 m Max. depth = 2.1 m
Side Slopes: first 3 m elsewhere	5:1 3:1	7:1 near normal water level, then 0.3 m steps	Maximum bank slope 5:1 between pond bottom and edge of pond at freeboard elevation

3.4.3 Incorporation of Water Quantity Control

According to MOE (2003), "many stormwater facilities are designed to meet multiple objectives (e.g. water quality, erosion control, quantity control). Such practice minimizes the land required for stormwater management and may eliminate the need for hydraulically operated flow splitters. In this case, the pond storage is increased by the volume required for quantity control and the outflow structure must include a weir that can pass the 100-year peak inflow routed through the pond.

3.4.4 Generic Design

Figure 10 and 11 show generic designs of an extended detention pond and its outlet configuration.

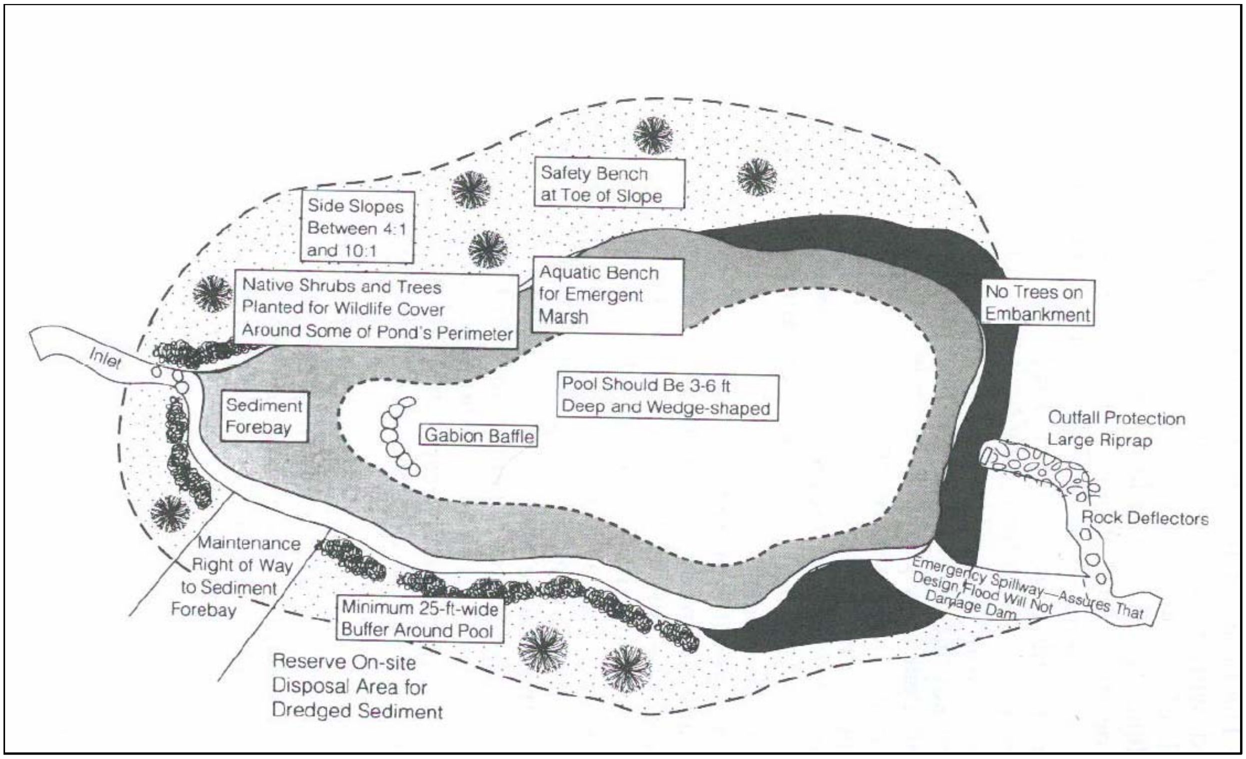


Figure 10 Generic Design of an Extended Detention Pond
(After WEF ASCE, 1992)

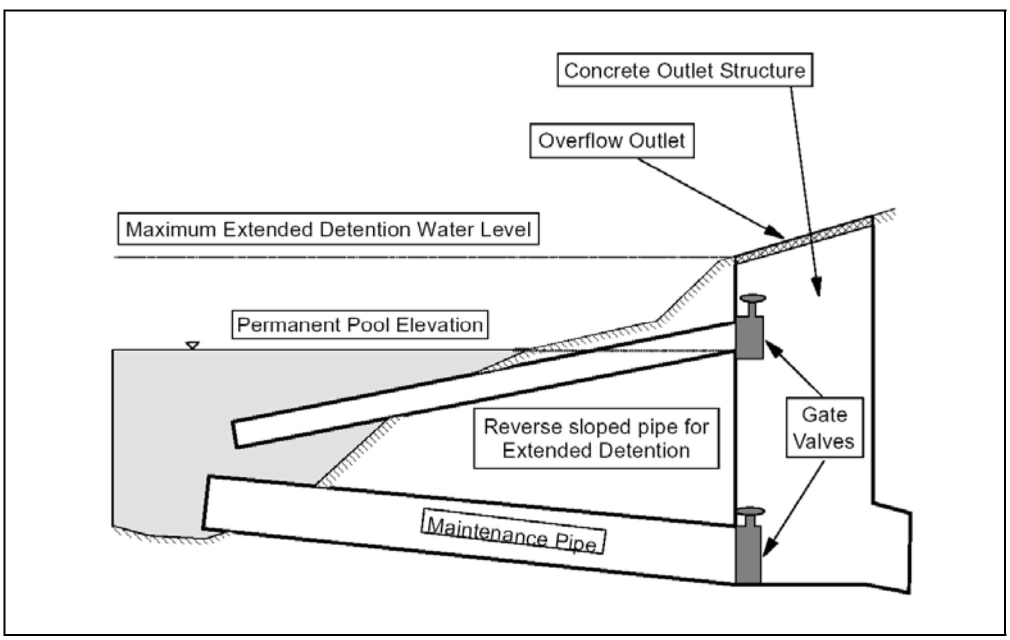


Figure 11 Generic Pond Outlet Configuration
(After MOE, 2003)

3.4.5 Summary

In all, 11 stormwater control facilities are identified. The sub basin to be developed, sub basin area and required storage volumes for each of these facilities are given in Table 3.4. In all cases, the water quantity storage governs in that it exceeds the active storage.

Table 3.4 Required Storages for Stormwater Control Facilities

Facility	Sub basin	Area (ha)	Impervious Level (%)	Permanent Pool Storage[1] (1000 m^3)	Active Storage[1] (1,000 m^3)	Water Quantity Storage[2] (1,000 m^3)
F1	4	151	21	9.7	6.0	32.3
F2	6	300	40	33.5	12.0	86.7
F3	8	182	61	30.2	7.3	79.8
F4	10	98	16	5.2	3.9	20.0
F5	12	123	35	12.3	4.9	34.0
F6	13	99	12	4.2	4.0	15.8
F7	14	110	49	14.9	4.4	34.6
F8	16	52	49	7.0	2.1	15.5
F9	17	93	18	5.4	3.7	12.1
F10	18	51	43	6.1	2.0	15.8
F11	19	57	41	6.5	2.3	16.4

[1] ***From Table 3.2.***
[2] ***From Table 3.1.***

Required storage comprises two components: a) permanent pool storage (required for water quality) and b) water quantity storage. Both components depend on the sub basin area and the level of development. In addition, water quantity storage depends on whether or not some of the flow is diverted out of the basin. The effect of sub basin area can be accounted for by considering storage per unit area (i.e. unit storage) in m^3/ha and level of development can be represented by “Impervious Level” in percent. Figure 12 shows the relations between the two components of storage and impervious level.

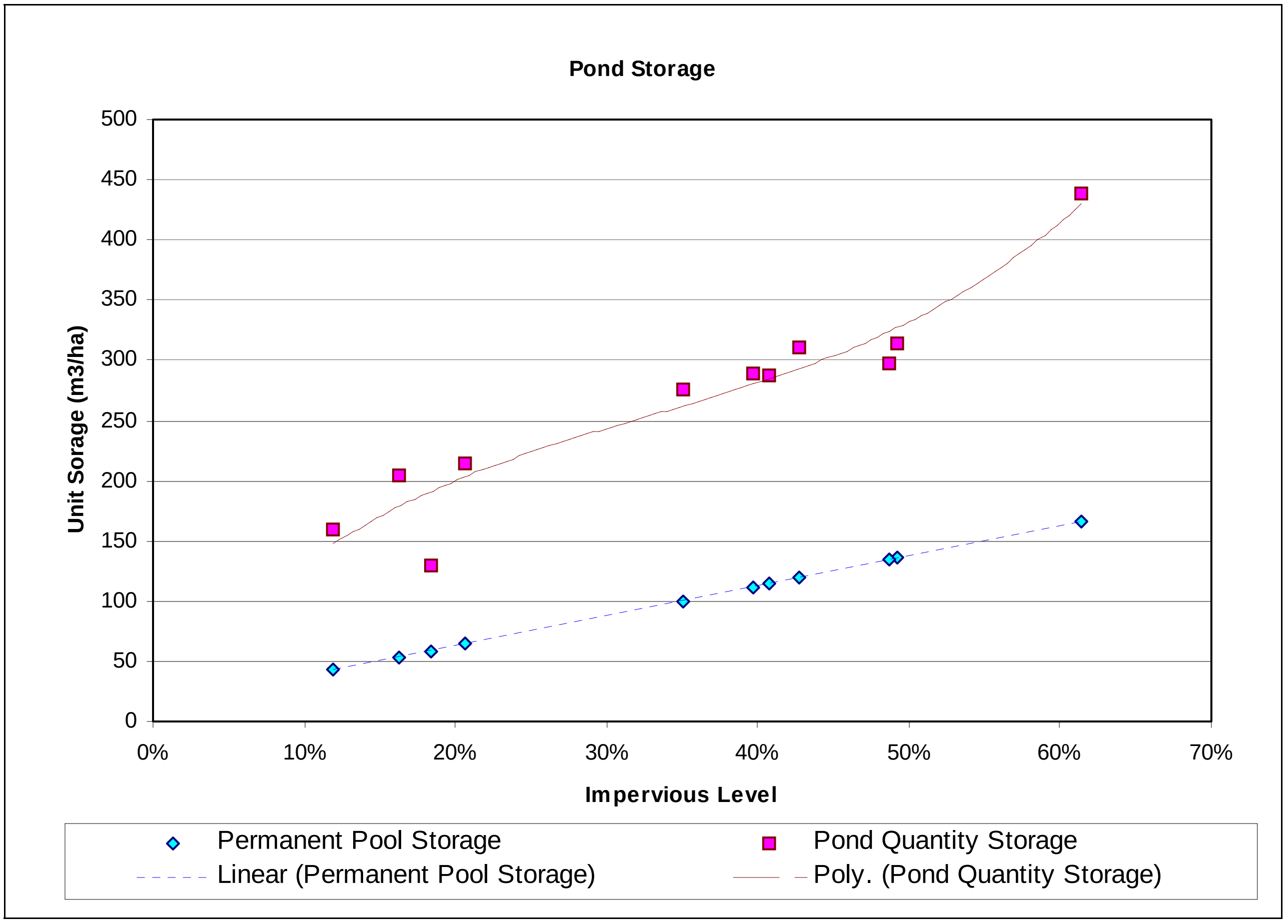


Figure 12 Pond Storage Relation

Over the range of impervious level from 12 to 61 %, permanent pool storage increases linearly with impervious level in a smooth fashion from 42 to 170 m3/ha.

Over the same range of impervious level, water quantity storage increases non-linearly with impervious level from 130 to 440 m^3/ha. The low outlier is sub basin 17, which has the lowest curve number (66). This fact, combined with an impervious level of 18 %, results in a lower value of water quantity storage. The high outlier is sub basin 8; the higher value of unit storage is due to the high value of imperviousness.

4. COST CONSIDERATIONS

4.1 General

Chapter 7 of MOE (2003) provides information on capital as well as operational and maintenance costs for stormwater management facilities. In terms of capital costs, Table 7.1 of the MOE document lists capital cost items (e.g. excavation, earthwork, etc.) that should be considered and Table 7.3 lists unit costs for each item. These tables can be used for estimating purposes once a proposed final design is available.

In terms of operation and maintenance costs, Table 7.5 provides unit costs for items ranging from litter control to sediment removal. This table can be used for estimating purposes once a proposed final design is available.

For the purposes of this study, we consider operation and maintenance (O&M) costs separately such that "regularly scheduled O&M costs" refer to costs that can be estimated with minimal uncertainty. Typically, these costs are for regularly scheduled maintenance items with maintenance intervals of one year or less. O&M costs associated with items that have intervals greater than one year where both the intervals and the costs have associated with them a large degree of uncertainty are termed "longer-term O&M costs". A typical item in this category is the cost of sediment removal and disposal. In this case, the interval and the cost depend on the sediment deposited, which in turn depends on the rate and type of development, sediment control during construction, and the design of the stormwater facility. These longer-term O&M costs are not typically part of an annual budget and sometimes are not even contemplated.

4.2 Capital Cost

4.2.1 Unit Capital Cost for Stormwater Ponds

Estimated construction costs in the Toronto area range from \$50 - \$60 per m^3 of design storage volume, for ponds with total design storage volume of 6,000 - 10,000 m^3 (unit costs for larger ponds are expected to be somewhat lower). This estimate includes all construction items including excavation, erosion control, outlet control structure, final grading and landscaping, but does not include any land acquisition costs. Also, it does not include engineering or contingency costs. Two recent estimates of pond costs (including land acquisition) in the Kingston area are \$47 and \$56 per m^3 of total storage, where the total storage values are 40,000 m^3 and 24,000 m^3 (lower unit cost for unit storage).

4.2.2 Cost Apportionment

There may be a need for a system to apportion costs for the case where different parts of a sub basin may be developed at different times or where different companies develop portions of a sub basin at the same time. In either case, the stormwater pond will be built at the time of initial development.

XCG suggests that costs be apportioned on the basis of impervious area for the following reasons.

i. Impervious area is most important contributing factor to the need for a stormwater treatment facility.

ii. Impervious area is easy to measure and not subject to interpretation.

We have estimated the values of post-development impervious area for each sub basin on the basis of the official plans for Belleville and Quinte West. However, we recommend that this calculation be updated at the time of first development in a sub basin using the best available information at that time. Then, for each development, the portion of the total cost will depend on the portion of the total impervious area for that development as set out in Table 4.1.

Table 4.1 Cost Apportionment

Portion of Total Impervious Area (%)	10	15	20	25	30	35	40	45	50	55	60
Portion of Total Pond Cost (%)	10	15	20	25	30	35	40	45	50	55	60

4.3 Maintenance Strategies

4.3.1 Contributing Factors

i. **Pond Design:** Maintenance costs can be reduced with a properly designed pond (i.e. sediment forebay inlet, length to width ratios, vegetative buffers, etc.).

ii. **Rate of Development:** Complete development of the watershed presents a challenge in terms of maintenance. There may be a need for a system to apportion costs (for the first clean-out) for the case where different parts of a sub basin may be developed at different times or where different companies develop at the same time.

iii. **Sediment Control:** High level of erosion and sediment control during construction is essential for reducing the time before the first cleanout is required.

iv. **Municipal Budget:** Experience has shown that access for the municipal capital budget is particularly challenging and hence an ongoing operating budget is generally preferable.

v. **Responsible Department:** Historically the costs for winter road maintenance (sanding/salting) and pond clean-out have been allocated to different municipal departments. Perhaps, the same department should be responsible for both activities.

4.3.2 Suggested Maintenance Strategy

i. **Delay of transfer to municipality:** The policy in Richmond Hill, a municipality responsible for over 80 stormwater ponds, is to delay transfer of the pond to the municipality until the developer has demonstrated that the pond functions as designed. This policy encourages the developer to design and construct acceptably performing ponds and to minimize erosion and sediment transport during construction.

ii. **Operating budget:** Because of the difficulty in securing funds through the capital budget, it is recommended that pond maintenance be added to the annual maintenance budget.

iii. **Linkages to road maintenance:** Because of the somewhat mutually exclusive goals of maximizing winter road safety and minimizing pond maintenance costs, it is recommended one department (the department responsible for roads and streets) be given responsibility of determining the optimum allocation and cost.

5. SUMMARY

XCG carried out the objectives of the Potter Creek stormwater drainage plan. Specifically:

1. Peak flows were determined for existing and post-development conditions for the AES 12-h, 100-year storm.
2. Storage requirements were determined at the outlets of all major sub basins, to maintain pre-development (existing) peak flows.
3. Water quality storage requirements were determined for all major sub basins using Ministry of Environment guidelines.
4. Guidance regarding basin-wide stormwater management measures was provided and a generic stormwater pond design was developed.
5. Guidance concerning costs, cost apportionment, and maintenance strategies were provided.

For modelling purposes, the most recent version of the US Corps of Engineers HEC-HMS was applied. Independent models were developed for pre-development, post-development without control and post-development with recommended control measures installed. Model parameterization made extensive use of available GIS information and the results of a recent LiDAR survey. Pond storage requirements, addressing both quantity and quality control in accordance with provincial guidelines, were derived for post-development conditions.

In addition to the AES 12-h 100-year rainfall event, the September 2004 rainfall event was applied as an input to the model for existing conditions. The 2004 rainfall event resulted in lower peak flows and runoff depths than the 100-year event. These model results are consistent with observations by QC staff that the maximum water level during the 2004 event, just upstream of Highway 2, was lower than the 100-year flood elevation.

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APPENDIX A
GIS DATA ACQUISITION

GIS Data Acquisition:

Using the LiDAR (Light Detection and Ranging) Digital Elevation Model developed for this project staff from Quinte Conservation's GIS Department extracted the following information using GIS software called ARCGIS 9.1 ArcHydro Extension;

- Delineation of the Potter Creek Watershed
- Delineation of the sub-basins within the Potter Creek Watershed (this process was then hand verified using 1 m contours and knowledge of existing culverts)
- Delineated the total contributing areas to junction nodes
- Created a soils map outlining the known soil types in each sub basin as shown on the County Soils mapping
- Delineated 1m contours for the entire Watershed
- Utilization of the ARCGIS software to determine the slope of each sub basin
- Utilization of the ARCGIS software to determine the length on each sub basin as well as the length of the longest flow path within each basin
- Created a Land Use map in conjunction with the Official Plans from both the City of Belleville and Quinte West

Additionally all the above procedures were duplicated using the Ministry of Natural Resources 10 m Digital Elevation Model to verify all delineated basins and sub-basins.

All GIS information was prepared for use within the hydrologic modeling being performed for the Master Drainage Plan for Potter Creek.

APPENDIX B

BELLEVILLE STORM WATER GUIDELINES

Guidelines for the Design and Construction
of Storm Water Management Facilities

The following guidelines provide the minimum requirements for the construction of Storm Water Management Facilities without security fencing. Where fencing is used, it shall consist of 1.8 metre high chain link fencing constructed in accordance with City specifications and provide a minimum of two (2) vehicle access gates for each enclosed pond.

Irrespective of these guidelines, the final decision as to whether a Storm Water Management Facility is to be fenced or not rests with the City of Belleville.

Dry Ponds

Storm Water Management Dry Ponds shall be designed to limit the maximum depth of water to 1.8 m above the lowest point of the storm water basin. An additional 0.3 m freeboard is required above the maximum peak flow flood level. The maximum depth of the extended detention zone shall not exceed 1.0 m above the lowest point of the pond.

Maximum bank slope shall be 5:1 and shall extend from the bottom of the pond to the edge of the pond at the freeboard elevation. The minimum allowable gradient on the bottom of the basin shall be 1.0% and the maximum gradient shall be 5.0%. The minimum horizontal distance between the bottom of the pond and the limit of maximum detention shall be 3.0 m.

Wetlands

Storm Water Management Wetlands shall be designed to limit the maximum depth of water to 2.1 m above the lowest point of the storm water basin excluding micropools. An additional 0.3 m freeboard is required above the maximum peak flow flood level. The maximum depth of the extended detention zone shall not exceed 1.0 m above the permanent pool elevation. Maximum peak flow attenuation zone shall not exceed 1.8 m above the permanent pool elevation. The permanent pool depth shall range between a minimum depth of 0.15 m to a maximum depth of 0.3 m.

Maximum bank slope shall be 5:1 between the bottom of the pond and the edge of the pond at the freeboard elevation. The minimum horizontal distance between the permanent pool elevation and the freeboard elevation shall be 3.0 m.

Micropools shall not exceed an additional maximum depth of 0.3 m below the permanent pool level and shall not exceed 5% of the total wetland permanent pool surface area.

Wet Ponds

Storm Water Management Wet Ponds shall be designed to limit the maximum depth of water to 3.3 m above the lowest point of the storm water basin. An additional 0.3 m freeboard is required above the maximum peak flow flood level. The maximum depth of the extended detention zone shall not exceed 1.3 m above the permanent pool elevation. Maximum peak flow attenuation zone shall not exceed 2.1 m above the permanent pool elevation. The permanent pool depth shall range between a minimum depth of 0.9 m to a maximum depth of 1.2 m.

Maximum bank slope shall be 5:1 between the bottom of the pond and the edge of the pond at the freeboard elevation. The minimum horizontal distance between the permanent pool elevation and the freeboard elevation shall be 3.0 m.

Forebays

Forebays are required for all of the above described Storm Water Management Facilities. The permanent pool depth shall range between a minimum depth of 0.9 m to a maximum depth of 1.2 m in which a maximum depth of 0.5 m shall be used for sediment accumulation. Forebays shall not exceed 33% of the total Wet Pond surface area and 20% of the Wetland permanent pool surface area. All other aspects regarding the design of Forebays shall conform to the above Wet Ponds standards. Excluding maintenance access routes, all access to Forebays shall be discouraged through shrub plantings.

General

1. The number of inlets/forebays shall be limited to one (1) where possible.

2. Wherever possible, pond inlet inverts from storm sewers shall not be lower than the maximum extended detention level in order to prevent surcharging of storm sewers upstream.

3. Maintenance access for Wetlands and Wet Ponds shall be accomplished by providing a 3.0 m minimum width gravel access road around the pond perimeter. This road shall be constructed with 150 mm of Granular B and 200 mm Granular A material. The maximum longitudinal gradient shall be 10:1. Where fencing is to be used a 4.0 m minimum width access road shall be constructed inside the fencing.

4. Where pedestrian access areas are used the maximum longitudinal gradient shall be 6:1 maximum.

General (cont'd)

5. Notwithstanding other provisions included herewith, it may be necessary in the design of headwalls to incorporate handrails or fencing at the discretion of the City because of water depth and/or steepness of bank slopes.

6. In all cases, implementation of the principles included herewith under these guidelines shall always have regard for approved Watershed, Sub-Watershed and Master Drainage Plans.

7. Areas subject to the collection of contaminants or spills shall be fitted with oil/grit separators satisfactory to the City.

8. New plantings shall be required for all ponds and forebays. Plantings shall be located within the area extending between the permanent pool elevation and the property line for Wet Ponds, Wetlands and Forebays. Plantings for Dry Ponds shall be located within the area commencing at a point 3.0 m from the bottom level of the pond and extending to the property line.

   Native and non-invasive trees, shrubs, ground covers (including grassed areas) and aquatic plants are required in a low maintenance landscape design which has regard for the ecology of the site and the eco-region.

   Trees shall be planted at a minimum rate of 1 tree (minimum 50 mm calliper) per 50 square metres. The density of shrub plantings for safety purposes shall vary depending on the degree of slope of the bank.

9. Wherever pedestrian access is located in close proximity to Storm Water Management Facilities, dense shrub plantings shall be placed adjacent to the pedestrian path to discourage access to the Facilities.

10. All fenced and non-fenced Storm Water Management Facilities shall be signed so that access is restricted to the general public. The number, location, size and wording of such signs shall be as directed by the City.

Subdivision Calculation for Cash in Lieu of Storm Water Management

Total Land Area of the Subdivision x.xx ha.

Total Impervious Area of the Subdivision		
Road Area	xxxx sq.m.	
Sidewalk Area	xxxx sq.m.	
House Area (Estimated)	xxxx sq.m.	
Driveways (Estimated)	xxxx sq.m.	
Total Impervious Area	xxxx sq.m.	= x.xx ha.

% Impervious = Total Impervious Area / Total Land Area = xx.x%

Cash in lieu amount = {$10,000 + $10,000(%Impervious)} x Total Land Area

Definitions

Road Area: The area of the asphalt pavement and concrete curb and gutter on the road allowance within the subdivision.

Sidewalk Area: The area of the concrete sidewalk and asphalt sidewalks or trails on the road allowance within the subdivision.

Estimated: Use the area of a typical house and driveway proposed for this subdivision and multiply by the number of lots.

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